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Stavridou, Nafsika; Baniotopoulos, Charalampos; Koltsakis, E.

NUMERICAL INVESTIGATION OF COLD-FORMED STEEL PROFILES AGAINST BUCKLING

Nafsika Stavridou[1], Efthymios. Koltsakis[2] and Charalampos C. Baniotopoulos[3]

[1,3]Department of Civil Engineering
University of Birmingham
Birmingham, B152EE, United Kingdom
e-mail: n.stavridou@bham.ac.uk ; web page: http://nstavrid.webpages.auth.gr/
e-mail: c.baniotopoulos@bham.ac.uk ; web page: http://ccb.civil.auth.gr/

[2]Department of Civil Engineering
Aristotle University of Thessaloniki
Thessaloniki, GR-54623, Greece
e-mail: mkolts@civil.auth.gr

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Abstract. *The most common support structure for on-shore wind energy converters has been the cylindrical steel tower with the lattice tower taking the lead when constructing taller structures. The components of the lattice structures have to sustain loads that go far beyond what industrial steel profiles used in lattice structures can carry. The present paper addresses the investigation of the behavior of custom made cold-formed L-shaped profiles against buckling. These profile types are selected since L-shaped profiles are very easy to mount and their buckling behavior is ameliorated with the introduction of brace lacing. The profiles investigated are of certain slenderness and with the introduction of various bracing motives the profiles are of class 3 and 4. Aiming to contribute to better understanding of the structural behavior of the L-shaped profiles, the present research work focuses on the development of reliable numerical models along with the use of analytical equations in order to predict accurately and interpret the structural response of the cross-sections against buckling. The bracing chosen based on structural and geometrical criteria is the V-shaped brace and specimens with varying brace density are analyzed and compared. From the comparison between the analytical and the numerical results valuable conclusions can be taken regarding the structural behaviour of L-shaped profiles with internal bracing and their applicability on wind turbine tower structures. The ultimate slenderness and bracing type can be selected and its application on wind structures can be tested. Based on the numerical results of the specimens the present investigation is further elaborated by the laboratory testing of selected specimens, which will result in the introduction of such cross-section in the construction of real wind turbine structures.*

1 INTRODUCTION

Global contemporary energy needs keep rising and climate change regulations make the use of sustainable energy means imperative in order to limit the atmospheric pollution. These two trends lead to the installation of more powerful and efficient power plants using sustainable energy sources. The European Commission has established a Renewable energy directive accepted by all member States, which sets as a goal that the final energy consumption from renewables should reach or even exceed 27% of the total energy consumption by 2030 [1]. Due to the high efficiency over the land occupied, wind energy is proved to be one of the most promising renewable energy sources and its broad expansion in Europe has been reflected on the fact that the total power capacity of installed wind power plants has tripled from about 50GW in 2005 to over 150GW in 2016 [2]. Towards enhancing the wind turbines' capacity, contemporary structures are designed taller in order to harvest the higher wind speeds that blow in greater distances from the earth's surface. When designing onshore wind converters, one can identify that the commonest tower configuration the cylindrical steel tower. Cylindrical towers consist of modules that are manufactured in the factory by cold-forming steel plates and transforming them to conical or cylindrical sections. The modules are subsequently transported and mounted on-site by means of bolted flanges [3]. In principal, cylindrical steel shells have due to their geometry, the general advantage of high strength to thickness ratio, meaning that they can sustain great loads with relatively small shell thickness.

As stated above, the current trends make the construction of taller turbines imperative in order to cover the higher contemporary needs for green energy. When designing supertall cylindrical towers though, certain structural problems arise that can be solved by either increasing the overall tower thickness or by introducing internal stiffeners while keeping the principal shell thickness constant [4] [5]. Keeping in mind that both solutions for enhancing the capacity of cylindrical shells are non-economic in terms of material use and adding on top that when constructing on shore wind farms on mountain tops certain transportation restrictions apply, a potential swift from cylindrical shells to lattice structures should be carefully considered. When constructing taller towers, the subparts tend to get longer along with the blades, which makes it difficult to transport by conventional trucks on the narrow and curvy roads usually found on mountain peaks. In addition, as wind turbine towers act as simple cantilever structures, when they get taller, the acting moment at the base becomes greater and therefore the bottom shell diameter has to be increased. Again, transportation of subparts with the conventional way of Lorries imposes a maximum limit for shell diameters to 3-4 meters, which is usually the maximum height of a vehicle that can pass through highway bridges. Finally yet importantly, the erection of wind turbine towers, requires the use of large-scale cranes which, when exceeding certain heights, are very expensive to hire and difficult to transport.

A solution to all the issues stated in the previous paragraph is attempted to be given through Hyper Tower Project where it is proposed to substitute tubular towers with lattice ones that are lighter and therefore easier to carry while the use of large-scale cranes is minimized since these lattice towers are self-rising. The conventional lattice solution has already been successfully implemented on large-scale towers on telecommunication masts. Contrary to wind turbines, telecommunication towers have no moving parts or great masses positioned at their top, which makes them easier to construct with standard L shaped cross-sections but the scale of the lattice towers which are able to support the rotor of a wind converter leads to cross sections that are well outside the range of standard industrial profiles. A lattice tower that is capable of accommodating the nacelle has the form of a truncated cone with a polygon or square cross-section. The implementation of lattice towers on either offshore or onshore wind turbines has just been the research field of numerous research groups [6] [7], and the optimal tower shape in terms of minimum material use has also been studied by the Hyper Tower research team. The lattice tower investigated and optimized is a statically determinate system composed of a number of discrete structural sub-systems; the legs, the face bracing trusses (FBT), horizontal braces and secondary bracings arranged inside the plane of the face bracing trusses. These discrete structural subsystems have a particular role in the load transfer mechanism of the lattice tower and since the tower is a statically determinate structure, the member deliver only axial stresses and the load that they carry can be determined by closed form expressions.

In the tower optimization process conducted, the square shaped lattice tower has been proved to be the optimal with each of the four faces comprising of the legs, the horizontal braces and the V-brace supporting system. In the overall tower investigation and for computational reasons, hollow tubular cross-sections were used to calculate the members of the tower [8]. When aiming to further minimize the overall steel mass used, the circular hollow sections can be substituted with angular members.

Single angular section members were the first structural shape broadly used as a traditional structural member due to the easiness in connection with other load carrying structural members [9]. This type of structural members has been extensively used in a variety of truss structures and the commonest connection detail leads to eccentric compressive loading. Eccentric axial compression combined with the asymmetric characteristics of the cross-section leads to a complicated and mixed torsional and flexural behavior [10]. This complex structural response has intrigued researchers and engineers in the past and the investigation of the non-linear behavior of single angular steel members has attracted the attention of the research community for a long time. Despite the general interest on the explanation of their structural behavior, it has not been but recently that the angular column non-linear behavior has been properly understood [11]. Several research groups have conducted experimental and numerical analyses in order to further investigate the structural behavior of symmetric angular members, which has led to the configuration of the design code guidelines [12].

The present work investigates the stability performance of a single angular member under axial compression and compares it with the structural response of an identical angular member where the open face is filled with a V bracing system. Both the unstiffened and stiffened members are investigated with the aid of finite element models and experimental work is yet to follow. The boundary conditions implemented are the ones that are planned to be applied on the laboratory tests and the loading conditions of the two specimens are identical.

2 NUMERICAL MODELING

In order to assess the structural behavior of angular steel members against buckling and the influence of the introduced bracing, two steel members are investigated. The first steel member is a simple angular cross section of 61.8 cm and the second member is an identical one with V-braces positioned to cover it’s initially open face as it is shown in Figure 1 and Figure 2 below. The bracing and the main specimen share the same thickness which is set to 5mm.

The cross-section investigated is a symmetric steel cross section of 81.36mm length for the outstand parts and the simple angular one belongs to class 4. According to Equation (1) of Eurocode EN 1993-1-1 [13] presented below, the slenderness of the angular member is 0.5.

$$\lambda = \sqrt{\frac{Af_y}{N_{cr}}} \tag{1}$$

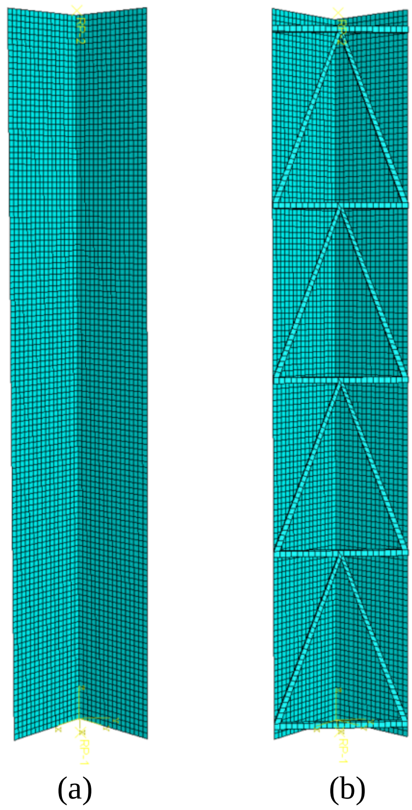


Figure 1. Numerical models for the unbraced (a) and the braced (b) specimen.

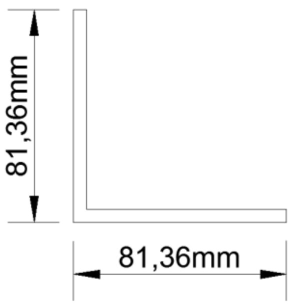


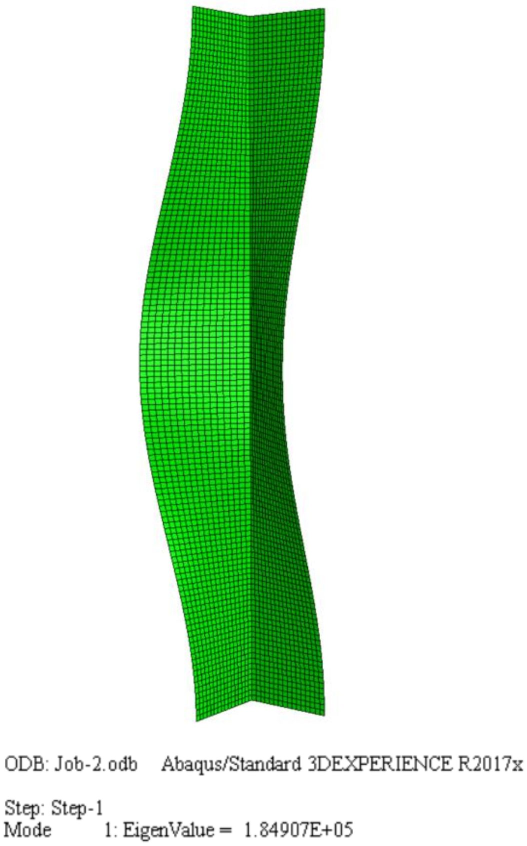
Figure 2. Plan of the simulated steel section.

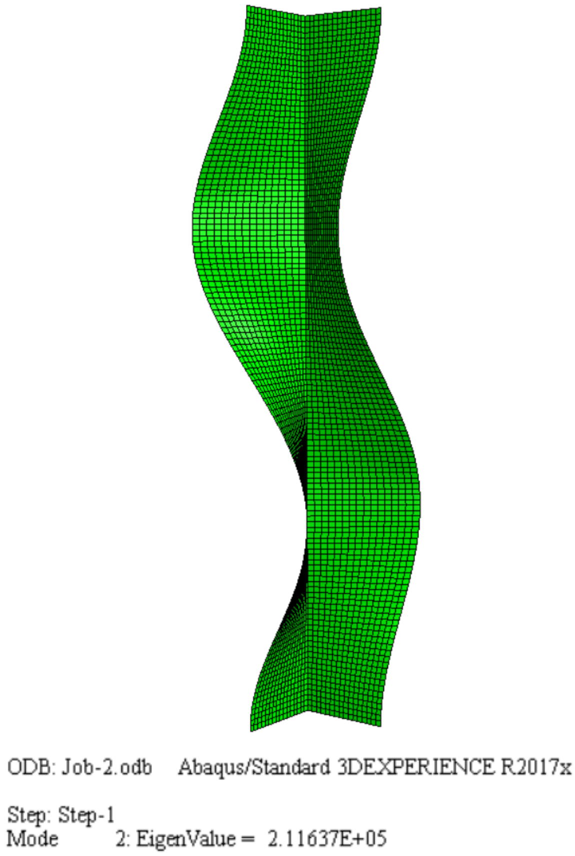
The simulation of the angular part of the steel members was realized with shell S4R shell elements as they are described in Abaqus software manual [14] while the bracing was simulated with solid elements of type C3D8R. The members were initially analyzed using the eigenvalue analysis, where the critical buckling load was calculated and the first three eigenmode shapes were obtained.

3 RESULTS

3.1 Eigenvalue Analysis Results

The unbraced specimen was first analysed, calculating the eigenmodes and the eigenvalues. The first three eigenmode shapes are presented in Figure 3 below, where the rotational buckling shape is evidently dominant. Torsional buckling is usually prevailing in non-symmetric steel members and especially in class 4 angular cross-sections. The eigenvalues which are interpreted in these analyses as the critical buckling load of the steel members are presented in Table 1 below.





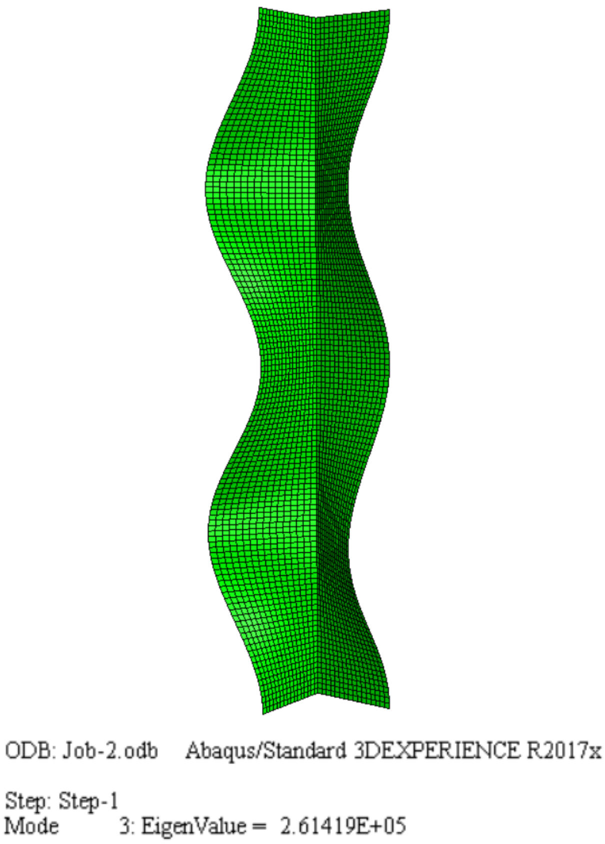


Figure 3. The first three eigenshapes and eigenvalues for the unbraced specimen.

	Unbraced member	Braced member
Eigenmode 1 (kN)	184.91	212.24
Eigenmode (kN)	211.64	250.744
Eigenmode 3 (kN)	261.42	325.133

Table 1: Eigenvalues of the Braced and Unbraced members.

The introduction of the stiffening scheme increases the critical buckling load of the member as predicted and changes also the buckling shape from torsional to flexural. This will be also evident later on in the static analysis.

3.2 Static Analyses

In order to assess the behaviour of the specimens against buckling two types of analyses have been implemented. First both the unbraced and the braced specimen were simulated and tested against compressive loads. The column is pinned at both ends with the boundary conditions and the loads acting on a reference point at the centre of gravity of the angular cross-section. After performing the static analysis for the unbraced and the braced specimen, the buckling behaviour was obtained as pictured in Figure 4 and Figure 6 respectively. The ultimate buckling load for the GMNA analysis for the unbraced and braced specimen is 178.52kN and 199.97kN respectively as presented in Table 2 below.

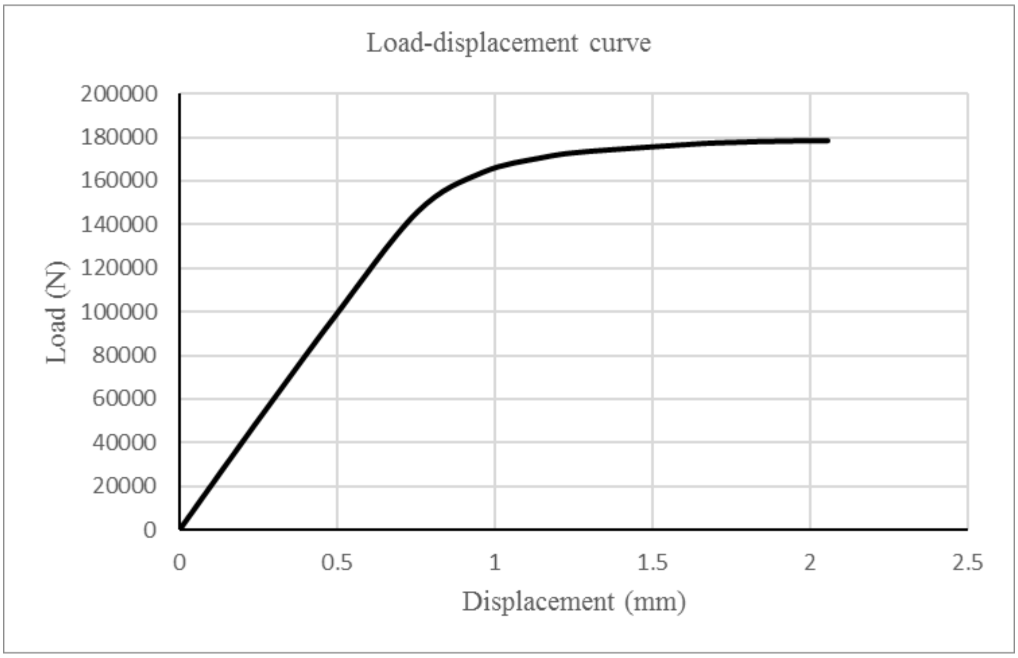


Figure 4. GMNA for the unbraced specimen.

After performing the GMNA analysis the influence of the initial imperfections has to be assessed. In order to take into account these specimen initial imperfections the technique implemented in the work of Speicher and Saal [15] is followed where the eignmodes of the specimens are initially calculated. The buckling shape of the first eigenmode is obtained and the appropriate amplification factor according to EN 1993-1-6 [16] is used to calculate the imperfection magnitude. For both cases of unbraced and braced specimen, the ultimate buckling load is lower when taking into account the initial imperfections as it noted in Table 2. The ultimate buckling load when performing Geometrical and Material non-linear imperfection analysis (GMNIA) is 173.90kN for the unbraced specimen and 193.03kN for the braced one. The load-displacement curves for both cases are calculated and presented in Figure 5 and Figure 7 below. The introduction of the bracing is not only increasing the ultimate buckling load of the specimen, but it is also changing the buckling shape of the specimen. While the torsional mode is dominant in the case of the unbraced specimen, the introduction of the bracing is converting the failure mode to a flexural one. This is pictured evidently in Figure 8, where the failure mode of the unbraced (a) with dominant torsion is presented in comparison with the flexural failure mode of the braced one (b).

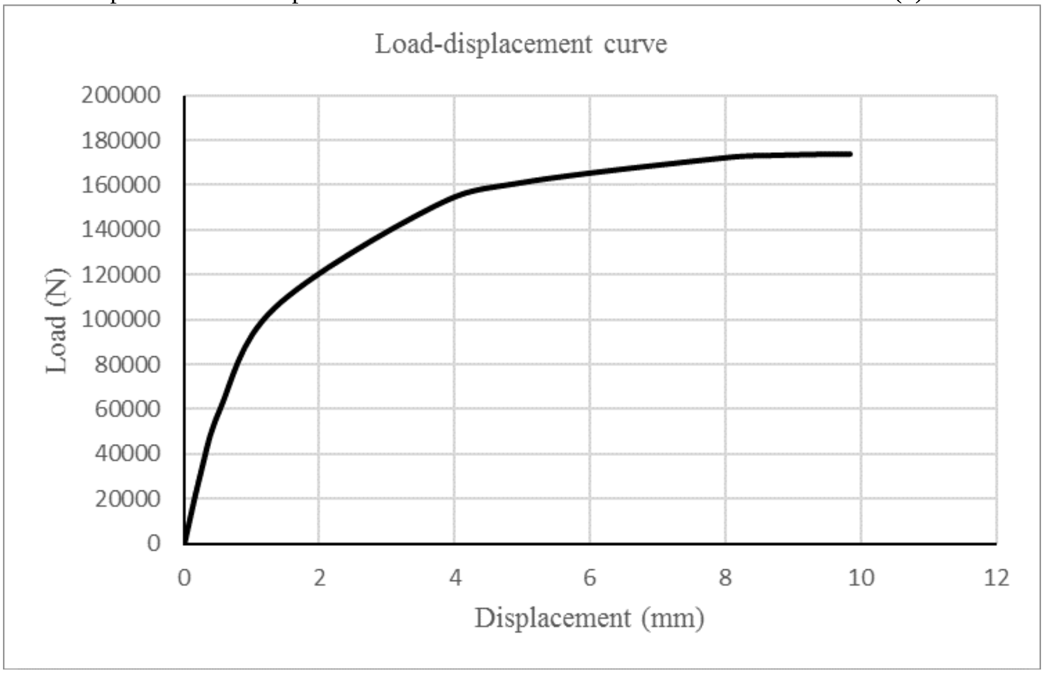


Figure 5. GMNIA for the unbraced specimen..

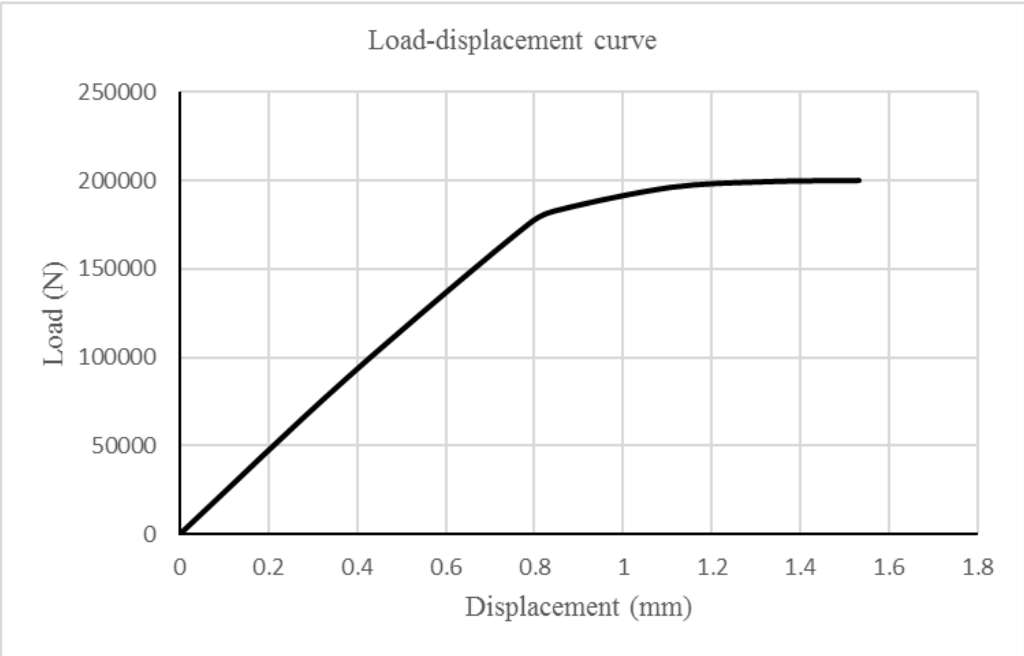


Figure 6. GMNA for the braced specimen.

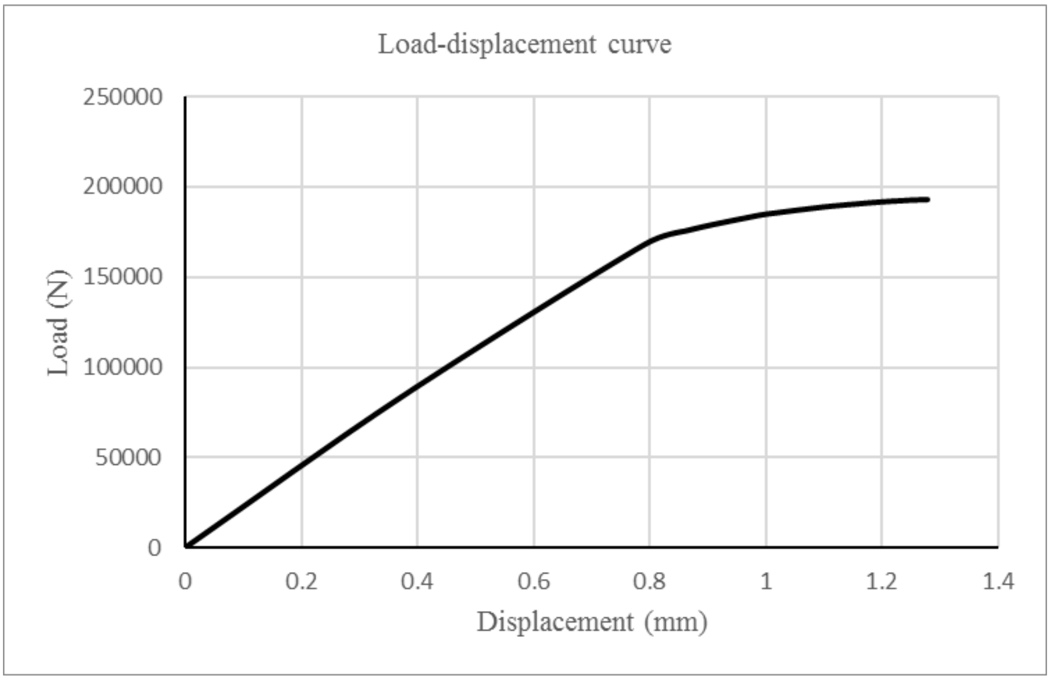


Figure 7. GMNIA for the braced specimen.

	Unbraced member	Braced member
GMNA	178.52	199.97
GMNIA	173.90	193.03

Table 2: Ultimate buckling load in kN for the anlayzes performed.

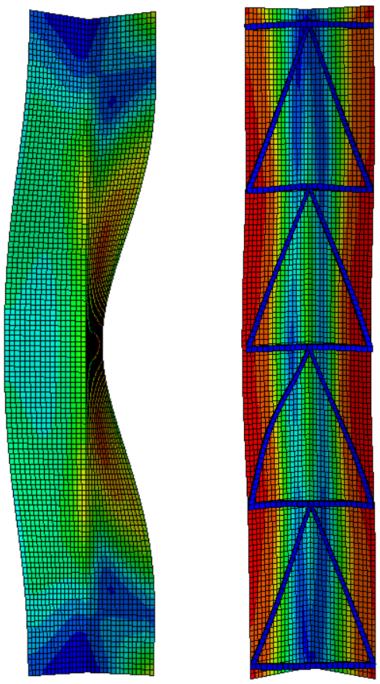


Figure 8. Plan of the simulated steel section.

4 CONCLUSIONS

The present work proposes the introduction of bracing in single angular steel sections to increase the ultimate buckling load of the specimen and more importantly to change the failure mode from torsional to flexural. The use of angular members is rather broad due to the ease in mounting and connecting with the other load carrying members. For non-symmetric cross-sections like the angular members, the importance of controlling the failure mode is rather important. In truss structures and more specifically on wind turbine towers, the structural members deliver exclusively axial loads and therefore it is important to be able to accurately predict the ultimate buckling load and the failure mode of the sections used. As far as the braced angular members are concerned, the change in the failure mode is evident in both GMNA and GMNIA analyses and the increase of the buckling load is 11-12% depending on whether initial imperfections are taken into account or not. The increase in the material used is almost 7%. This particular behaviour refers to member with slenderness close to 0.5 and further research is being carried out in order to verify the numerical analysis results with laboratory experiments, and with specimens of varying slenderness.

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